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THE DYNAMIC AND EARTHQUAKE RESPONSE OF A TWO STORY OLD R/C BUILDING WITH MASONRY INFILLS IN LIXOURI-KEFALONIA, GREECE INCLUDING SOIL-FOUNDATION DEFORMABILITY.

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Abstract. *The dynamic and earthquake behaviour of a two-story reinforced concrete(R/C) frame building with masonry infills located in Lixouri-Kefalonia-Greece is examined. This building was constructed according to old seismic code provisions and was subjected to intense earthquake ground motion during the recent 2014 earthquake activity in the island. Despite the intensity of the seismic action this structure developed light to moderate damage that is confined mainly to the masonry infills and to the spalling of the unconfined concrete cover of the columns. The numerical simulation includes influences arising from the masonry infulls that are within the R/C frames as well from the R/C foundation mat that is resting on relatively flexible soil. Use is made in the numerical simulation of the deformability of the foundation based on the results obtained from a specific investigation that dealt in measuring the dynamic characteristics of a bell tower located at a close distance from the examined building. The response of the masonry infills is also included employing a methodology that takes into account various non-linear mechanisms that develop during the infill-R/C frame interaction when this type of structural system is subjected to horizontal seismic forces. In addition, the possibility of the development of plastic hinges at predetermined locations of the reinforced concrete elements is also included in the numerical investigation. Through the study of the obtained numerical predictions an effort is made to understand the observed performance of this building. The obtained numerical predictions of its earthquake response demonstrate that the minimum observed structural damage in this case may be attributed to the favourable influence of the well built masonry infills as well as of the continuous foundation mat interaction with the flexible soil without adverse effects.*

1 INTRODUCTION

The objective of this paper is to study the dynamic and earthquake response of a two-story reinforced concrete(R/C) structure that houses the City Hall of the city of Lixouri in the island of Kefalonia, Greece. This structure has two particular points of interest. The first point is that its ground floor R/C foundation mat hosted an instrument which recorded the acceleration of the damaging earthquake of 3rd February, 2014. The second point is that this structure is at a 350m distance from the bell tower of Agios Gerasimos (figures 1 and 2).

Figure 1. View of the center of Lixouri

Figure 2. The two-story R/C building that housed the City Hall of Lixouri prior to the seismic event of 3rd February 2015. Because of the developed structural damage is not occupied at present.

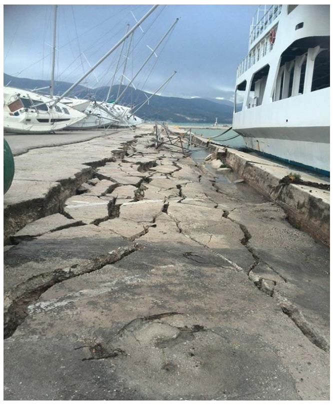


Figure 3. The earthfill embankement at the harbor immediately after the seismic event of 3rd February 2015

This bell tower has been the subject of in-situ dynamic measurements in order to quantify the influence of the soil-foundation deformability at this site[14]. Moreover, spectacular damage was observed at the earth fill embankment that forms the harbor of Lixouri (shown at the bottom of figure 1) as well as the overturning of boats that were free standing at the promenade that was at a relatively close distance from the City Hall building (figure 3). The in-situ study of the effects of strong earthquake ground motion gives the basis to check the validity of methods of analysis as the one attempted here [1, 2, 8, 13]. Use is made in this study of the measured acceleration of this particular strong ground earthquake motion.

The island of Kefalonia has been subjected to intense earthquake activity for a long time [2]. The whole region near Lixouri was subjected during the winter of 2014 in an intensive earthquake sequence and this gave the opportunity to focus, among other structures, on the earthquake response of the City Hall building as well as this bell tower which remained unscathed, despite the intensity of the seismic ground motion and the damage of neighboring structures [14]. In figure 1 the location of the City Hall of Lixouri is shown together with the bell tower of Agios Gerasimos at a distance of 0.35km where in-situ tests for measuring its dynamic characteristics were performed ([14]). It is interesting to note that the intensity of shaking during this damaging 3rd of February 2014 seismic event was recorded by a strong motion accelerograph located at the ground floor of the City Hall building (figures 4 to 6).

Figure 4. Ground floor location of accelerograph

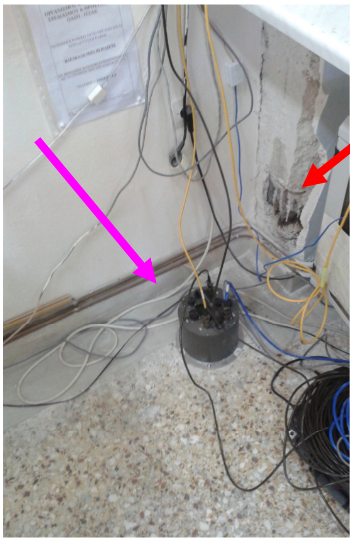


Figure 5. Ground floor location of accelerograph

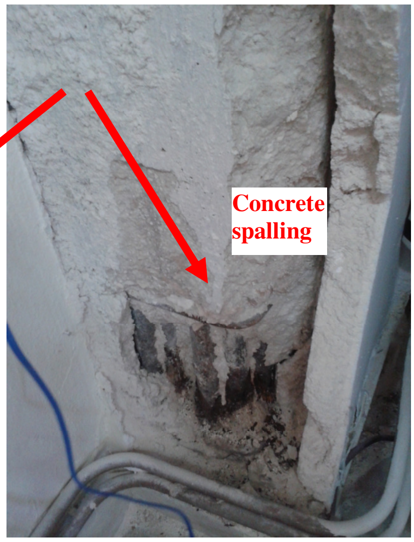


Figure 6. Signs of distress at the toe of nearby R/C column

Figure 7 depicts the plan of the ground floor and the 1st story of the City Hall at Lixouri. The total length in the longitudinal North-South (x-x) direction is equal to 24.3m whereas the total length in the transverse East-West (y-y) direction at the North side is equal to 16.54m and at the South side 11.07m. The mid-plane of the R/C slab that serves as the ground floor ceiling is located at 4.24m from the ground floor level (top surface of the foundation slab) whereas the mid-plane of the R/C slab that serves as the 1st story ceiling is located at 9.19m from this ground floor level. The structural system in the longitudinal North-South (x-x) direction is characterized by relatively short length R/C beams, approximately 4m long, that are joined monolithically with the adjacent columns. On the contrary, in the transverse East-West (y-y) direction the structural system is formed by a number of single bay two-story R/C frames. The bays of these frames have a clear span of approximately 8.40m from the mid-axes of their columns. Moreover, as can be seen in figure 7 the columns of these two-story R/C frames are 300mm x 500m and the relevant beams are 300mm wide and 700mm deep. Most of these frames in the upper 1st story are without any masonry infills in the transverse East-West (y-y) direction, thus creating a considerable clear space which was utilized for the City Hall meetings (figure 8). On the contrary, brick masonry infills occupy most of the bays of these R/C frames at the ground floor, thus creating various separations to be utilized as office space.

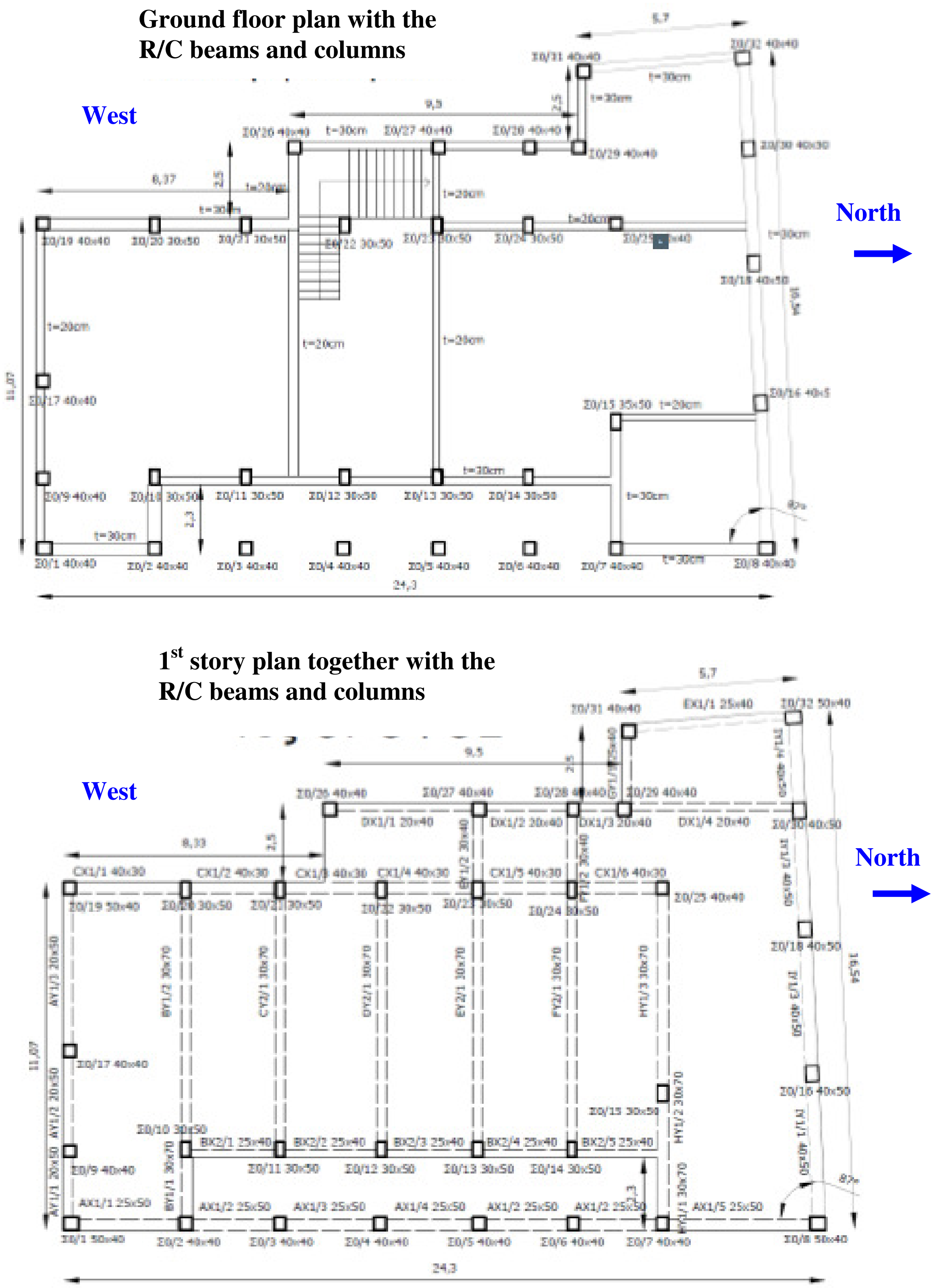


Figure 7. Plan of the ground floor and the 1st story of the City Hall at Lixouri

Figure 8. Reinforced concrete frames in the upper 1st story at the East-West y-y direction. The clear space was utilized for the City Hall meetings. Visible signs of masonry infill R/C frame interaction. Exterior wall.

Masonry infills occupy a large number of bays of the R/C frames that are created at all the external sides of this structure. These sides facing East, West and North (figure 7) have door and window openings whereas the side facing South is fully built with masonry infills as it is almost in contact with the adjacent building.

The most visible damage of the City Hall due to the 3rd of February 2014 strong seismic ground motion was in the interior and exterior masonry infills (figures 8, 9 and 10). Signs of distress at the toe of certain interior and exterior R/C columns at the ground floor level could also be observed (figures 6 and 11, respectively).

Figure 9. Visible signs of masonry infill R/C frame interaction. Interior wall .

Figure 10. Visible signs of masonry infill R/C frame interaction. Exterior walls .

Figure 11 Signs of distress at the toe of an exterior R/C column

As can be concluded from the material presented here one point that is worth being investigated is the influence of the presence of masonry infills in the dynamic and seismic response of this building. Significant research effort has been devoted in the past in the study of the interaction between masonry infills and the surrounding R/C frames during earthquake excitations [4, 5, 6, 7, 9, 10, 11, 12]. These studies have investigated this problem experimentally as well as numerically in trying to understand the most significant response mechanisms. The experimental results were utilized to validate analytical and numerical tools capable of predicting the response of R/C structures with frames hosting relatively strong of masonry infilled frames [9, 10, 12]. Such tools will be employed also in this present work as will be explained in the following sections. The second point of interest is investigating the influence of the foundation deformability. As already mentioned, from various observation in the Lixouri area it becomes apparent that the soil foundation deformability may have exerted certain influence in the observed dynamic and seismic response. This became evident from the in-situ measurements of the dynamic response of the Agios Gerasimos bell tower ([]). Therefore, it was decided to study this soil-foundation deformability in a parametric way as will be explained in the following sections.

.Finally, the presence of a R/C staircase that leads from the ground floor to the 1st story it is a third point of interest, which is also investigated. In summary the following parameters are studied:

a1. The soil-foundation deformability, **a2.** The presence of the masonry infills, **a3.** The presence of the staircase.

2 SOIL – FOUNDATION DEFORMABILITY

The foundation of this building is a R/C continuous slab with a thickness approximately equal to 0.5m. It is already mentioned that in-situ studies were performed with the Agios Gerasimos bell tower in order to research influences on the dynamic and earthquake response arising from soil-foundation deformability [14]. In order to study numerically the soil-foundation deformability the following process was utilized.

First, a numerical model of the R/C foundation slab was formed as shown in figure 12. Together with the foundation slab, which was modeled with shell elements having linear elastic properties based on the Young's modulus of concrete, the soil layers underneath this slab were also modeled being in full contact with the R/C slab. These soil layers extended to a depth of 4m and were formed with linear elastic solid brick elements.

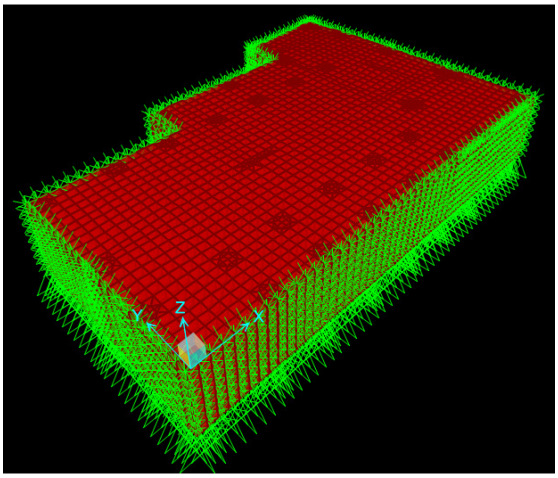


Figure 12. Numerical model of the foundation slab and the underlying soil.

Because it was not possible to find reliable geotechnical data in the vicinity of the studied structure the investigation of the soil-foundation deformability was attempted in a parametric way as will be explained below in order to define the elastic properties of the soil elements.

1st case. A shear wave velocity equal to 450m/sec was assumed for the underlying soil. This value, together with a soil density equal to 20KN/m3 leads to a shear modulus equal to 416.7MPa. This value together with a Poisson's ratio value equal to 0.2 leads to a Young's modulus value for the soil equal to 1000MPa. This represents a rather hard soil.

2nd case. Following the same rational but this time assuming a shear wave velocity value equal to 200m/sec leads to a the Young's modulus value for the soil equal to 196MPa. This represents a medium stiffness soil and is comparable with the corresponding value found from the Agios Gerasimos in-situ measurements.

3rd case. This time the shear wave velocity value was set equal to 150m/sec that represents a rather flexible soil that leads to a Young's modulus value for the soil equal to 110MPa.

Next, the top surface of the foundation slab was loaded in the vertical direction with a uniformly distributed pressure equal to -1.0MPa, with the positive vertical axis being upwards (figure 12). The bottom surface of the bottom layer of the soil volume was constrained to have zero displacements in all three directions. The side boundaries of the soil volume were constrained only in the two horizontal directions. In this way the foundation slab was displaced almost uniformly in the vertical direction being compressed in this way. The following table 1 lists the resulting vertical deformation of the slab for the three cases of soil deformability being considered.

Table 1. Vertical deflections of the soil-foundation simulation

Soil deformability	1st case Vs=450m/sec	2nd case Vs=200m/sec	3rd case Vs=150m/sec
Vertical slab deflection (mm)	3.6mm	18mm	33mm

In order to simplify the final numerical model of the City Hall structure including the soil-foundation deformability the following approximation was made. An alternative numerical model of the foundation slab was formed. This model retained the R/C foundation slab simulated with shell elements. However, the soil layers being simulated before with solid brick elements were replaced with two-node 3-D link elements having one node fully restrained in the all three directions as the support to the earth and the other node in full contact with each node of the foundation slab. Moreover, these two-node links were given such an axial stiffness that when the foundation slab was compressed with the same surface pressure as before (1.0MPa) the resulting vertical deflection of the foundation slab remained the same as before. Moreover, in certain cases a non-linearity was introduced in the axial direction of these two-node link elements resulting in their inability to sustain any tension. Following this rational the axial stiffness of these two-node link elements is listed in the following table 2.

Table 2. Axial stiffness of the equivalent two-node links

Soil deformability	1st case Vs=450m/sec	2nd case Vs=200m/sec	3rd case Vs=150m/sec
Axial Stiffness of the vertical two-node links (KN/mm)	55	10.5	5.9

3 THE PRESENCE OF THE MASONRY INFILLS

Use is made here of an equivalent post-elastic "pushover" type of analysis that draws information on the stiffness and strength variation from one-bay, one-story R/C masonry infilled unit frames that compose a given multistory structural formation [9, 10, 11, 12]. This numerical simulation is based on the well known substitution of the masonry infills with equivalent multi-linear diagonal struts. However, in order for this substitution to be realistic the following procedure is adopted, as is briefly described below.

a1) The multi-story structural formation is decomposed to a number of individual single-story masonry infilled frame units that are grouped according to their common geometric and mechanical characteristics of the R/C elements and their masonry infills, in order to minimize in this way the number of different units to be analyzed in the next steps (steps b1 to e1).

b1) For each one of these single-story masonry infilled R/C frame units a fully non-linear simulation is performed in order to obtain through a "pushover" type of analysis the full non-linear response for each individual single-story masonry infilled R/C frame composing the multi-story structural formation. This simulation, which is described in detail in [10] and [12] includes:

- The numerical simulation of the masonry infill utilizing non-linear plane stress finite elements.
- The numerical simulation of the surrounding R/C frame with linear elastic beam and column members together with predetermined locations of possible plastic hinge formation at the ends of each element.
- The numerical simulation of the interface between the R/C frame and the masonry infill with 2-D non-linear joint elements in the axial and transverse direction.

c1) For each one of these single-story masonry infilled R/C frame units a "pushover" type A analysis is performed and the horizontal load (H) versus horizontal displacement (δ) or shear strain (γ) response curve is obtained.

d1) For each one of these single-story masonry infilled R/C frame units a number of alternative numerical simulations are next prepared. Each one of these alternative simulations assumes that the R/C members of the surrounding frame retain their capability of developing plastic hinges at their ends as in b1) above whereas the masonry infill is replaced with an equivalent multi-linear diagonal strut member with properties as explained in e1) below.

e1) The alternative numerical analysis is now performed for the same single-story units of step d1; the objective here is to obtain horizontal load (H) versus horizontal displacement (δ) or shear strain (γ) response for a number of inter-story drift values corresponding to a number of pre-selected shear-strain levels for the masonry infill (e.g. 0.1%, 0.15%, 0.2%, 0.25%, 0.3%, etc.). The alternative analysis response results should be in reasonable agreement with the response values obtained through the fully non-linear simulation for the same infill masonry single-story frame unit. This is achieved through a trial and error approach by adjusting properly in the alternative analysis the multi-linear properties of the diagonal strut that replaces the masonry infill for each one of the single-story masonry infill R/C frame units obtained initially by n the fully non-linear simulation in step c1.

f1) Next, all the masonry infills in the numerical simulation of the multi-story structural formation are replaced with such multi-liner equivalent struts

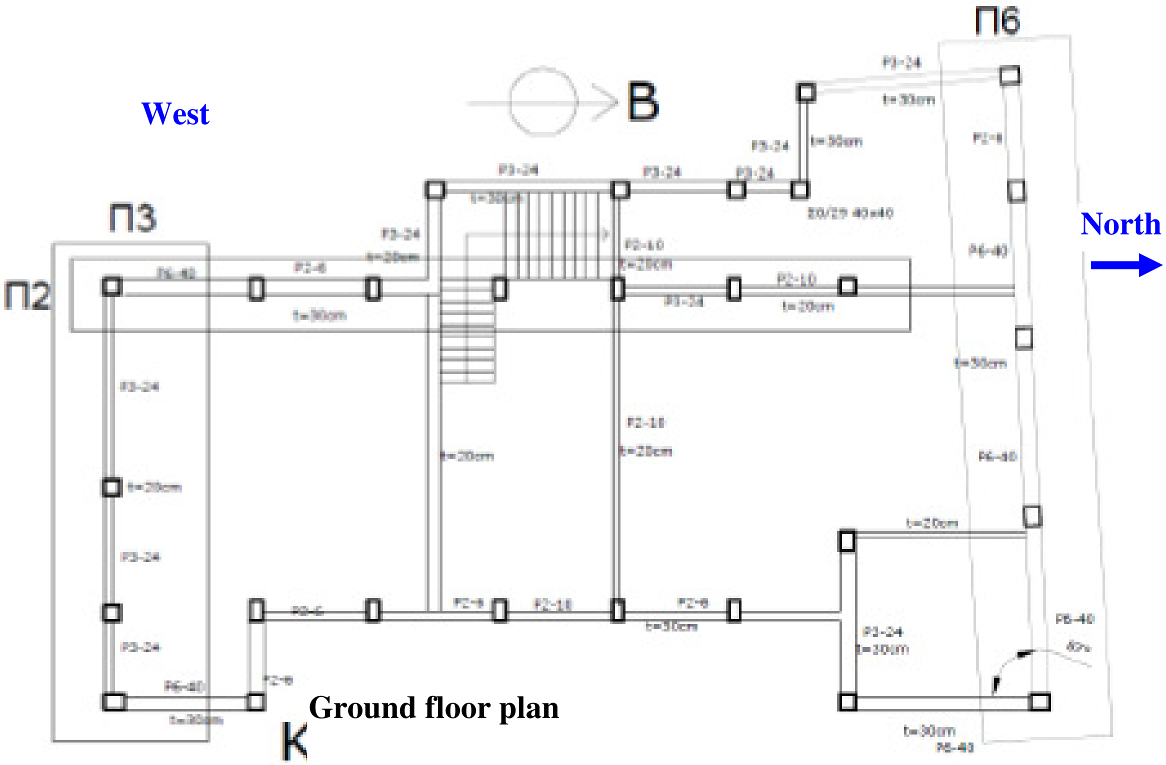


Figure 13 Ground floor plan with the indications of the masonry infilled "unit" frames designated with their code names: P2-10, P2-6, P3-24, P6-40

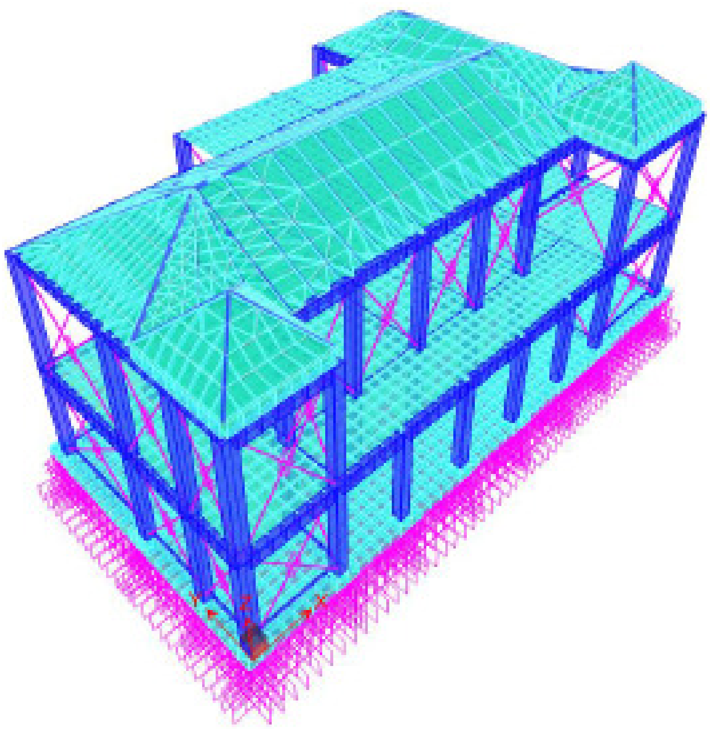


Figure 14 3-D representation of the City Hall building with the diagonal struts representing the masonry in-fills in place for the ground floor as well as for the 1st story.

Figure 13 depicts the ground floor plan where the various single-story one-bay infilled frames are denoted with a code name such as P3-24 for the infilled of the South side and most of the West side, P6-40 for the infilled frames of the North side and some of the East side, P2-6 the remaining infilled frames of the East side and P2-10 an infilled frame of the interior in the East-West direction and some remaining infilled frames at the exterior of the City Hall R/C building. The same process is followed for the infilled frames of the first story (figure 14). These single-story one-bay infilled R/C frames represent the "unit" infilled frames that this multi-story building is decomposed to. Following the numerical process described before the properties of multi-linear equivalent diagonal struts are found for each one of these the "unit" infilled frames. These diagonal struts are represented in the numerical simulation with two-node links having axial stiffness properties found in the way described before (steps a1 to f1).

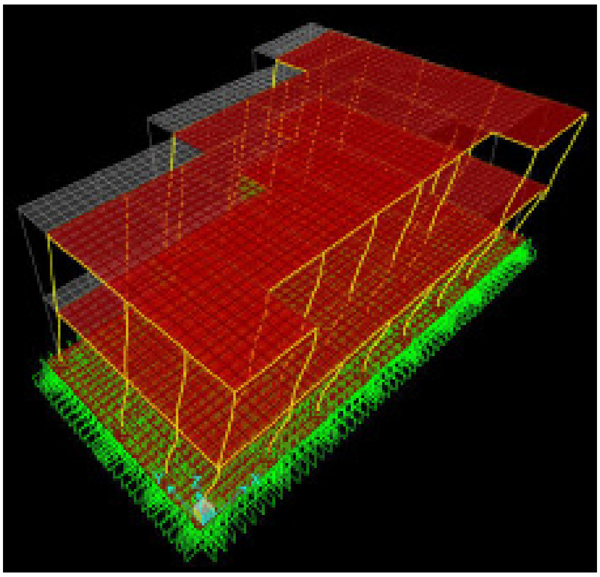


Figure 15. Numerical model of the City Hall building without masonry infills

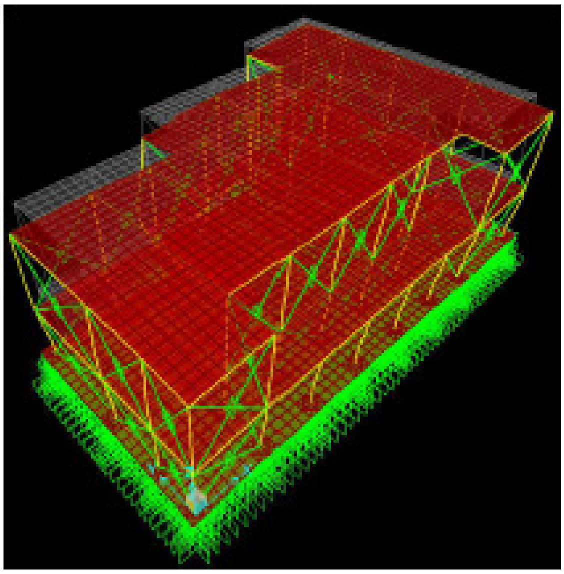


Figure 16. Numerical model of the City Hall building without masonry infills

For the linear elastic analyses the masonry infill in each bay was represented by two diagonal struts (figure 16) having 50% of the stiffness of the single "equivalent" diagonal strut defined in the way described before (steps a1 to f1). In figure 15, the numerical simulation of the City Hall R/C building without masonry infills is also shown. The following table 3 lists (2^{nd} row) the stiffness value of the first branch of these equivalents struts that is valid for the linear numerical analyses.

Table 3. Axial stiffness value for the employed diagonal struts

	Code Name of "Unit" infilled frame and Used initial stiffness values KN/mm			
	P2-10	P2-6	P3-24	P6-40
Linear case 50% of or the single "equivalent" diagonal strut. Two struts per bay	85.2	19.6	60.9	79.0
Non-linear case 100% of or the single "equivalent" diagonal strut. One strut per bay	170.3	39.2	121.8	158.0

For the non-linear push-over type of analyses these multi-linear links were defined in a way to be able to sustain only compression and no tension with initial stiffness equal to the full 100% of the value found for the single equivalent" diagonal struts (Table 3 3rd row). Moreover, only a single two-node link was provided per "unit" frame joined diagonally with the surrounding frame nodes in a way as to develop compression with the given stiffness value up to a level of inter-story drift beyond which each link was assumed to yield as shown in the following figures .

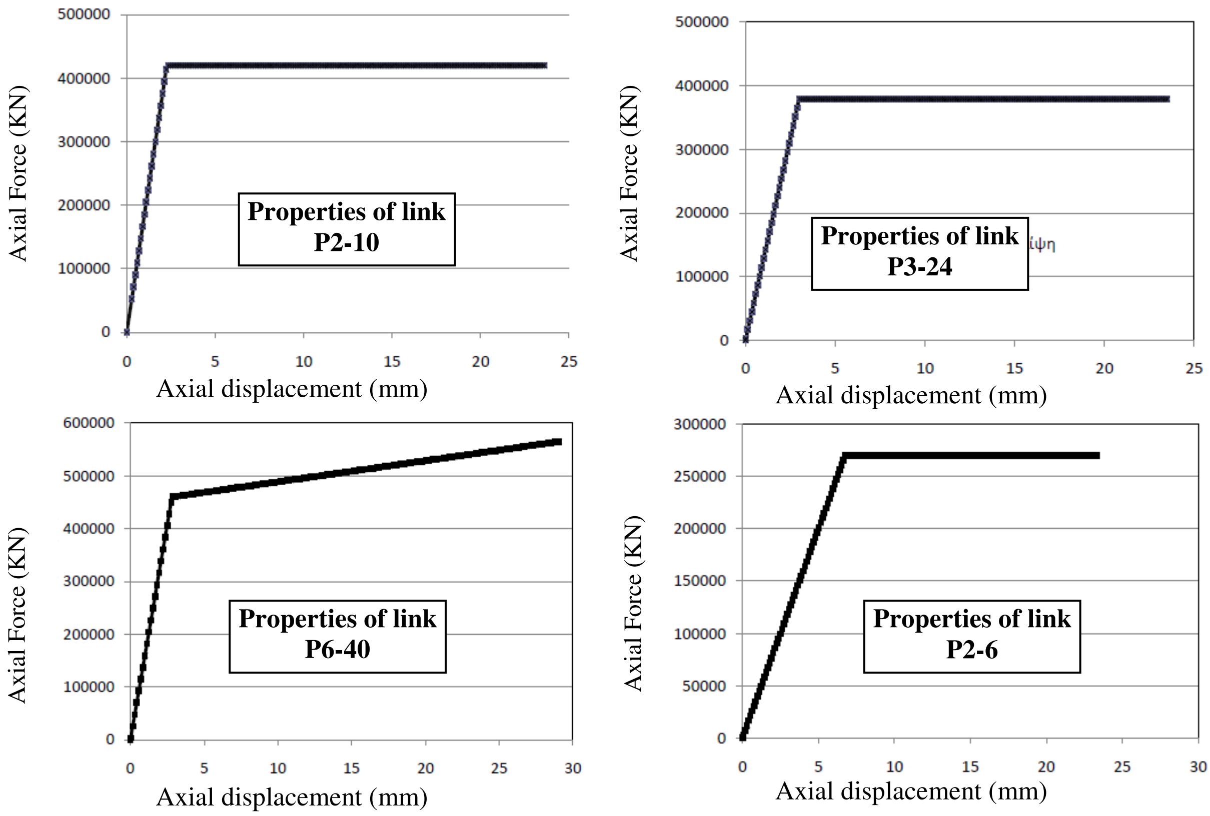


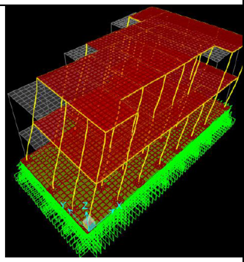
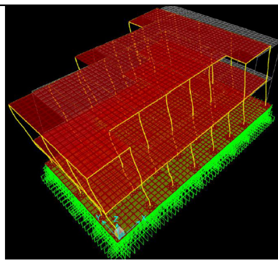
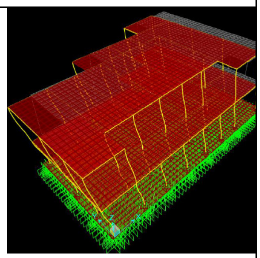
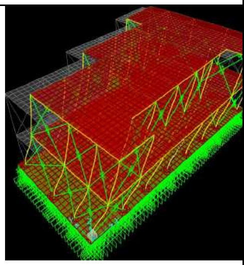
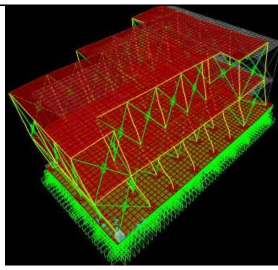
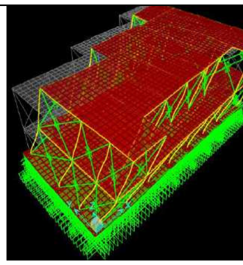
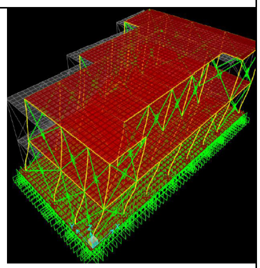
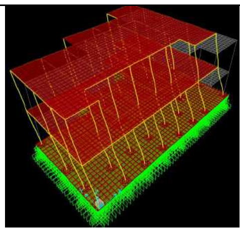
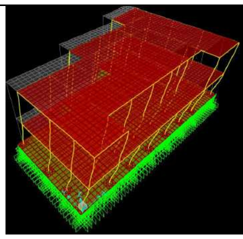
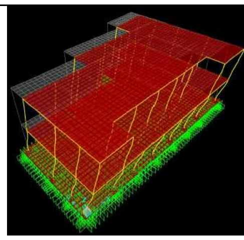
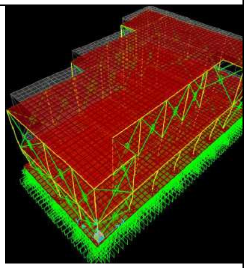
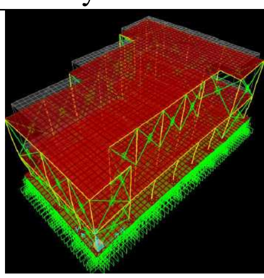
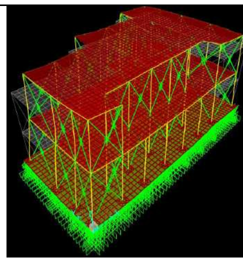
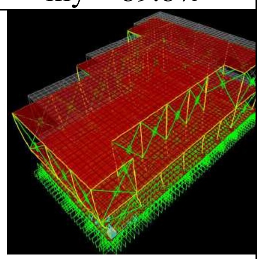
Figure 17. Axial load – axial displacement properties of the nultilinear two-node links representing the equivalent diagonal struts as a numerical approximation of the maonry infills.

4 THE LINEAR DYNAMIC RESPONSE OF THE CITY HALL BUILDING

The eigen-modes and eigen-periods of the City Hall building are investigated next in a parametric way, assuming linear elastic behaviour of all the elements, including the diagonal struts and the foundation links. The following parameters are varied:

a2) The superstructure is considered either with the diagonal struts representing the masonry infills or without these struts, thus numerically simulating the "bare" superstructure only with the reinforced concrete structural elements (slabs, beams and columns). These structural elements are the same in the numerical simulation without and with masonry infills when the diagonal struts are added with properties as outlined in section 3 (figures 15 and 16)
b2) The soil-foundation deformability is approximated in the way described in section 2. That is a 1st case is examined having vertical two-node links at the soil-foundation slab interface with axial stiffness equal to 55KN/mm, representing relatively hard soil conditions. Then, a 2nd case is examined having vertical two-node links at the soil-foundation slab interface with

axial stiffness equal to 10.5KN/mm, representing a soil of medium deformability. Next, a 3rd case is examined having vertical two-node links at the soil-foundation slab interface with axial stiffness equal to 5.9KN/mm, representing a rather flexible soil. Finally, a 4th case is examined whereby the foundation slab is presumed to be rigidly attached to the soil-foundation interface.

Figure 18	Soil-Foundation deformability 1st eigen-period T1			
	1st case hard soil	2nd case medium soil	3rd case Flexible soil	Non-deformable Soil
Bare superstructure without masonry infills North-South (x-x)	 T1=0.474sec mx = 83.8%	 T1=0.480sec mx = 83.8%	----	 T1=0.462sec mx = 87.6%
Superstructure with masonry infills (two diagonal struts per bay) North-South (x-x) East-West (y-y)	 T1=0.214sec mx = 53.9% my = 31.6%	 T1=0.229sec mx = 45.0% my = 39.2%	 T1=0.237sec mx = 34.1% my = 48.9%	 T1=0.202sec mx = 23.8% my = 64.8%
	Soil-Foundation deformability 2nd eigen-period T2			
	1st case hard soil	2nd case medium soil	3rd case Flexible soil	Non-deformable Soil
Bare superstructure without masonry infills North-South (x-x)	 T2=0.379sec my = 86.5%	 T2=0.386sec my = 86.2%	----	 T2=0.366sec my = 89.8%
Superstructure with masonry infills (two diagonal struts per bay) North-South (x-x) East-West (y-y)	 T2=0.209sec mx = 35.8% my = 52.3%	 T2=0.220sec mx = 43.7% my = 44.0%	 T=0.227sec mx = 53.9% my = 33.7%	 T2=0.20sec mx = 71.6% my = 23.6%

The variation of the two structural formations (with and without masonry infills) and the four soil-foundation deformability conditions results in (8) different numerical models of the City Hall building. In figure 18 the 1st and 2nd eigen-modes are depicted together with the corresponding eigen-periods and the mass participation ratios mx and my. The following remarks can be made on the basis of the results presented in figure 8.

1) As was expected, the addition of masonry infills increases drastically the stiffness of the structural system, and therefore reduces considerably the corresponding eigen-period values, despite the mass increase that accompanies these masonry infills. It is well known that up to the point these masonry infills remain undamaged, and thus active in terms of in-plane stiffness, the resulting structural formation is considerably stiffer than the corresponding structure that either it was not being built with such masonry infills or the infills become eventually inactive by being damaged during the earthquake sequence.
2) An additional influence that becomes apparent from the eigen-periods values listed in figure 18 together with the corresponding mass participation ratio values is the fact that the structural system of the City Hall building, when it is considered as “bare” that is without the masonry infills, vibrates in its two fundamental eigen-modes in a mainly translational way, either in the North-South or in the East-West direction. The addition of masonry infills results in the two fundamental eigen-modes becoming coupled in the North-South and East-West direction. This must be attributed to the fact that the location and the stiffness of these masonry infills in the City Hall building is not symmetric thus resulting in this coupled dynamic response.
3) The influence of the soil-foundation deformability is less drastic. Again, as was expected, the more flexible soil-foundation condition results in a modest increase in the fundamental eigen-periods values.
4) The increase in the soil-foundation deformability does not have a noticeable influence on the mass participation ratio values for the two fundamental eigen-modes when the structural system is without masonry infills (“bare” structure). The addition of the masonry infills apart from resulting in a coupled translational response in the North-South and East-West direction also has a noticeable influence on the corresponding mass participation ratio values.

5 PREDICTIONS OF THE EARTHQUAKE RESPONSE FOR THE CITY HALL BUILDING

Next, numerical simulations of the earthquake response of the City Hall building are presented. The numerical analyses that were performed towards obtaining earthquake response predictions of the City Hall building fall in the following two main categories.

5.1 Static numerical analyses with non-linear mechanisms for the two-node links at the foundation-soil interface and for the diagonal struts

In the first category a non-linear step-wise analysis is followed. For this type of numerical analyses the first step represents the application of the vertical loads; these loads result from the load combination D+0.3Q, where D represents the permanent vertical loads and Q represents the variable vertical loads. The horizontal seismic loads are next applied stepwise at the levels of the slabs of the 1st story floor and the 1st story ceiling either in the North-South (x-x) or in the East-West (y-y) direction. The ratio of the amplitude of these horizontal forces is kept constant in all steps and is set equal to the ratio of the corresponding horizontal displacements that resulted from the modal analysis at the story mass centers in the horizontal direction (either x-x or y-y) being considered. These are obtained for each particular structural formation, resulting from the combination of the superstructure with or without masonry infills and of the soil-foundation deformability variations, from the modal dynamic analyses presented in section 4. Two types of non-linearity are introduced in this first category of numerical seismic analyses; the first is the one described in section 2 for the two-node links representing the soil-foundation deformability; that is that these links are unable to sustain any tension. The second type of non-linearity is the one related to the behaviour of the diagonal struts representing the masonry infills, as described in section 3; this time these diagonal struts are assumed to yield at a certain level of axial deformation (figure 17). For this 1st category of numerical seismic analyses the R/C members are considered not to yield. The maximum seismic load that is applied in each case is defined according to the following rational.

It is presumed that the seismic loads could be defined according to the acceleration response spectral values derived from the acceleration measurements at the top surface of the foundation slab of this building. These are depicted in figures 19 and 20 for the North-South (x-x) and East-West (y-y) directions, respectively.

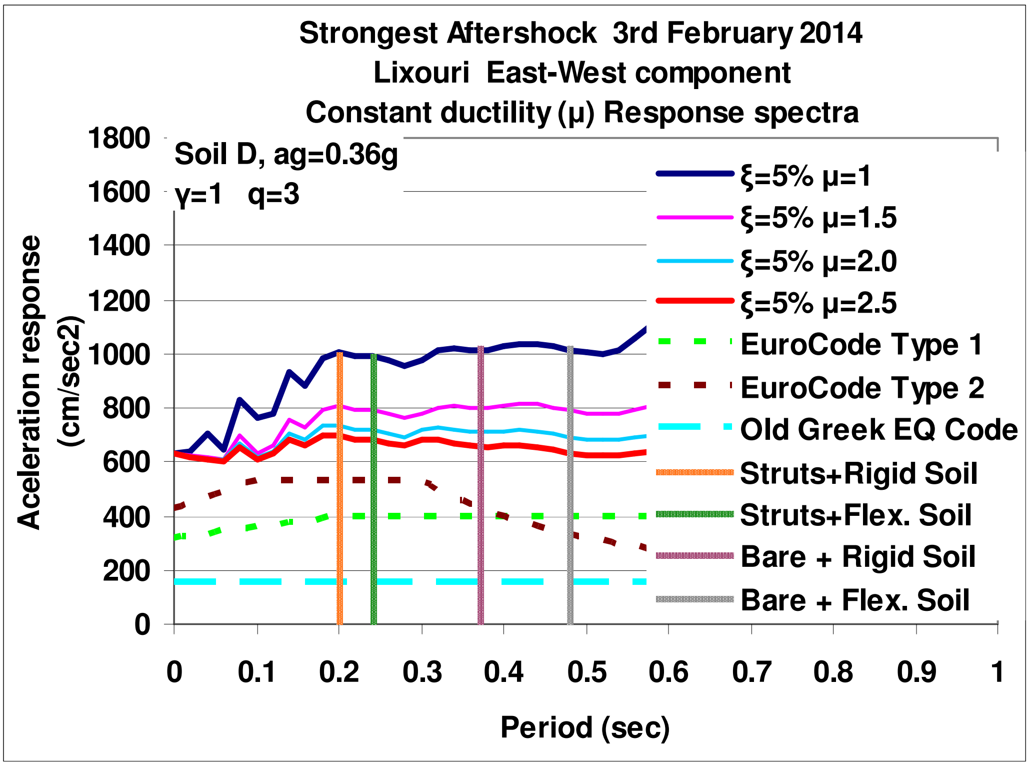


Figure 19. Definition of East-West (y-y) seismic loads through the recorded ground motion

The horizontal seismic loads in the y-y (E-W) direction are applied through the Lixouri East-West elastic acceleration response spectral curve for 5% damping ratio (figure 19, q=1) as it was derived from the E-W component of the acceleration record that was recorded during the strong aftershock of 3-2-2014 ([1]). This was recorded by an instrument located at the ground floor slab of a R/C two-story building (City Hall) located at a distance of 350m from the bell tower, as indicated in figure 1 (se also figures 4 to 6).

The response spectrum values have not being scaled to account for a response modification factor. The reason for this is the fact that, apart from the visible damage of the masonry infills, that has been described in the introduction (figures 7 to 9), the main R/C structural elements are with relatively limited signs of distress at the toe of few interior and exterior R/C columns at the ground floor level (figures 6 and 11). As can be seen, the acceleration spectral values for 5% damping and response modification value (q) equal to 1 are quite high at the level of the acceleration of gravity (g=981cm/sec2). In comparison, the "Old Greek Seismic Code" level of seismic forces that one presumes that this building was designed for are approximately 5 times less than these acceleration spectral values [3]. If this building would have been designed today according to the Euro-code 8 [15] design specrtra (either type 1 or 2) and for a response modification factor value equal to q=3 (typical for this type of R/C frame building) the corresponding force levels would be one half (1/2) of the force levels resulting from the recorded ground motion response spectra values shown in figure 19. Under these considerations, it is rather surprising that the structural damage of this building is at so minimal levels. This issue will be further discussed on the basis of the numerical predictions. It is interesting to underline here that the various eigen-periods, as they resulted from the eigen-mode study that was presented in section 4, correspond to almost the same acceleration spectral value for 5% damping and q=1 for all the various structural configurations considered in this study (with or without diagonal struts and rigid or flexible soil conditions).

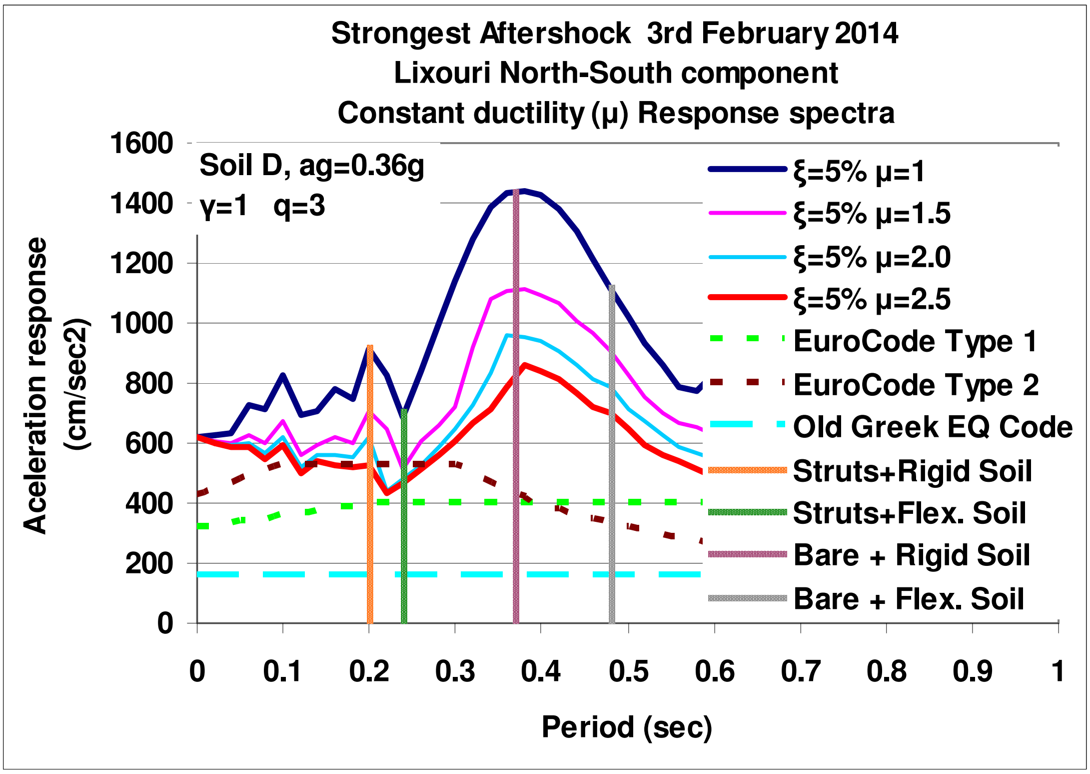


Figure 20. Definition of North-South (x-x) seismic loads through the recorded ground motion

As explained before, the horizontal seismic loads in the x-x (N-S) direction are applied through the Lixouri North-South elastic acceleration response spectral curve for 5% damping ratio (figure 20, q=1) as it was derived from the N-S component of the acceleration record that was recorded during the strong aftershock of 3-2-2014, [1] (figure 1).

The response spectrum values have not being scaled to account for a response modification factor, as explained before. As can be seen the acceleration spectral values for 5% damping and response modification value (q) equal to 1 are very high, reaching 1.3 times the acceleration of gravity (g=981cm/sec2) in case the masonry infills were not present or approximately 80% of the acceleration of gravity with the masonry infills present and before serious damage could render them inactive. In comparison, the “Old Greek Seismic Code” level of seismic forces, that one presumes that this building was designed for, are approximately 6 to 4 times less than these acceleration spectral values [3]. If this building would have been designed to-day according to the Euro-code 8 [15] design specrtra (either type 1 or 2) and for a response modification factor value equal to q=3 (typical for this type of R/C frame building) the corre-sponding force levels would be 70% of the force levels resulting from the recorded ground motion response spectra values shown in figure 20. Under these considerations, it is again ra-ther surprising that the structural damage of this building is at so minimal levels. This issue will be further discussed on the basis of the numerical predictions. It is interesting to under-line here that the various eigen-periods, as they resulted from the eigen-mode study that was presented in section 4, correspond to much larger spectral acceleration values for the N-S re-corded ground motion when the structural system is considered without masonry infills (“bare” structure) than when the masonry infills are present and not yet inactive by being damaged.

The following table lists the base shear values (at the top surface of the foundation slab) as they resulted from a linear elastic dynamic spectral analyses utilizing the East-West or North-South acceleration response spectral curves for 5% damping and q=1 derived from the re-corded ground acceleration (figures 19 and 20).

	Base shear Values (KN)			
	“Bare” structure without masonry infills and Medium soil	Structure with masonry infills and Rigid foun-dation	Structure with masonry infills and Medium soil (2^{nd} case)	Structure with masonry infills and Flexible soil (3^{rd} case)
North-South (x-x) direction	7180	6179	5209	4844
East-West (y-y) direction	6756	6448	6036	6046

5.2 Push-over numerical analyses with non-linear mechanisms for the two-node links at the foundation-soil interface, for the diagonal struts and all the R/C members

In the 2^{nd} category of numerical seismic analyses the non-linear behaviour of all the R/C members is also considered on top of the non-linearity of the two-node soil-deformability links and of the non-linearity described for the diagonal struts representing masonry infills. Thus, an additional set of non-linearity is introduced at all the ends of the R/C elements (rep-resenting either R/C beams or R/C columns) in an effort to simulate numerically the devel-opment of flexural plastic hinges at these locations when the flexural demand rises beyond certain amplitude. This type of numerical seismic analysis was performed considering the seismic loads and response only in the East-West (y-y) direction. This time the numerical analysis has again a first step when the vertical loads from the load combination D+0.3Q are introduced. Next, a “push over” type of horizontal loads are introduced at the 1^{st} story floor and the 1^{st} story ceiling levels. These horizontal seismic loads are applied at the center of mass of these slabs and result from the product of the corresponding story masses with the same acceleration amplitude that is gradually being increased at each subsequent step. The seismic horizontal displacement attained at each step is checked at the center of mass of the 1^{st} story ceiling.

6 NUMERICAL PREDICTIONS OF THE SEISMIC RESPONSE

6.1 Results from the static numerical analyses with non-linear mechanisms for the two-node links at the foundation-soil interface and for the diagonal struts

In order to present the obtained results in a summary form the following process is followed. First, the flexural demands in terms of maximum/minimum bending moment values, as they are obtained from the 1st category of numerical analyses are found at the top and foot of all the columns as well as at the ends of all beams. This is done for all the studied numerical simulations of the structure with or without masonry infills and for the four cases of soil-foundation deformability. In what follows, earthquake response predictions from the structure with diagonal struts (presence of masonry infills) and for three soil-foundation deformability (rigid, medium and flexible soil) are presented. Moreover, the seismic loads are applied both in the North-South / South-North (±x-x) and East-West / West-East (±y-y) directions.

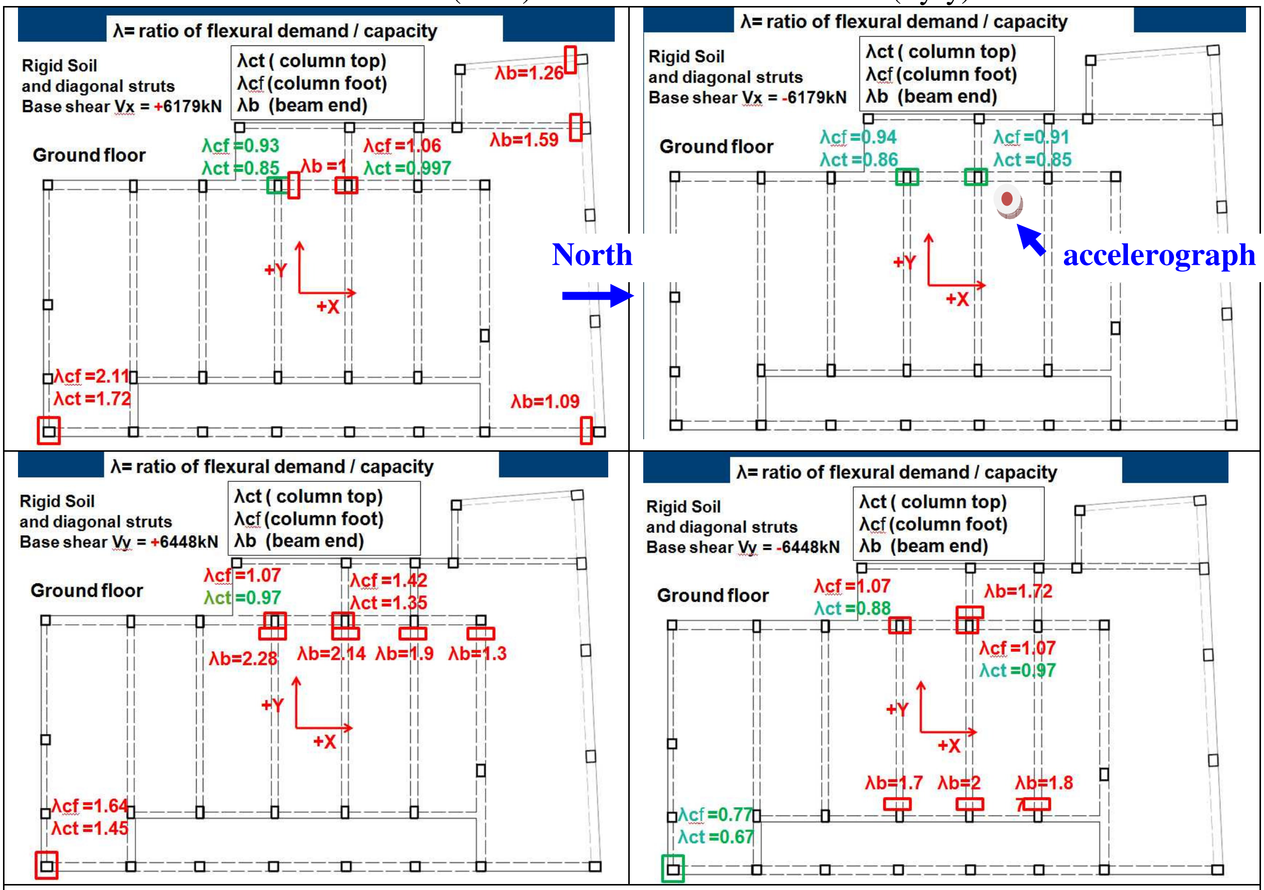


Figure 21. Flexural demand/capacity ratio values of R/C elements at the ground floor. Structure with diagonal struts and rigid soil conditions.

These results are presented in figures 21, 22 and 23 for the soil-foundation deformability cases that correspond to rigid, medium and flexible soil conditions, respectively. Instead of presenting the flexural demands by themselves these demands are compared with the corresponding flexural capacities at the same location and the ratio values (λ) of flexural demand / capacity are finally plotted in figures 21 to 23. The flexural capacity of the cross-sections was found by employing RCCOLA.NETv0.972 assuming the concrete strength equal to 20MPa and the steel yield stress equal to 385MPa. Apart from checking the various R/C members in flexure the shear capacity was also checked but is not reported here.

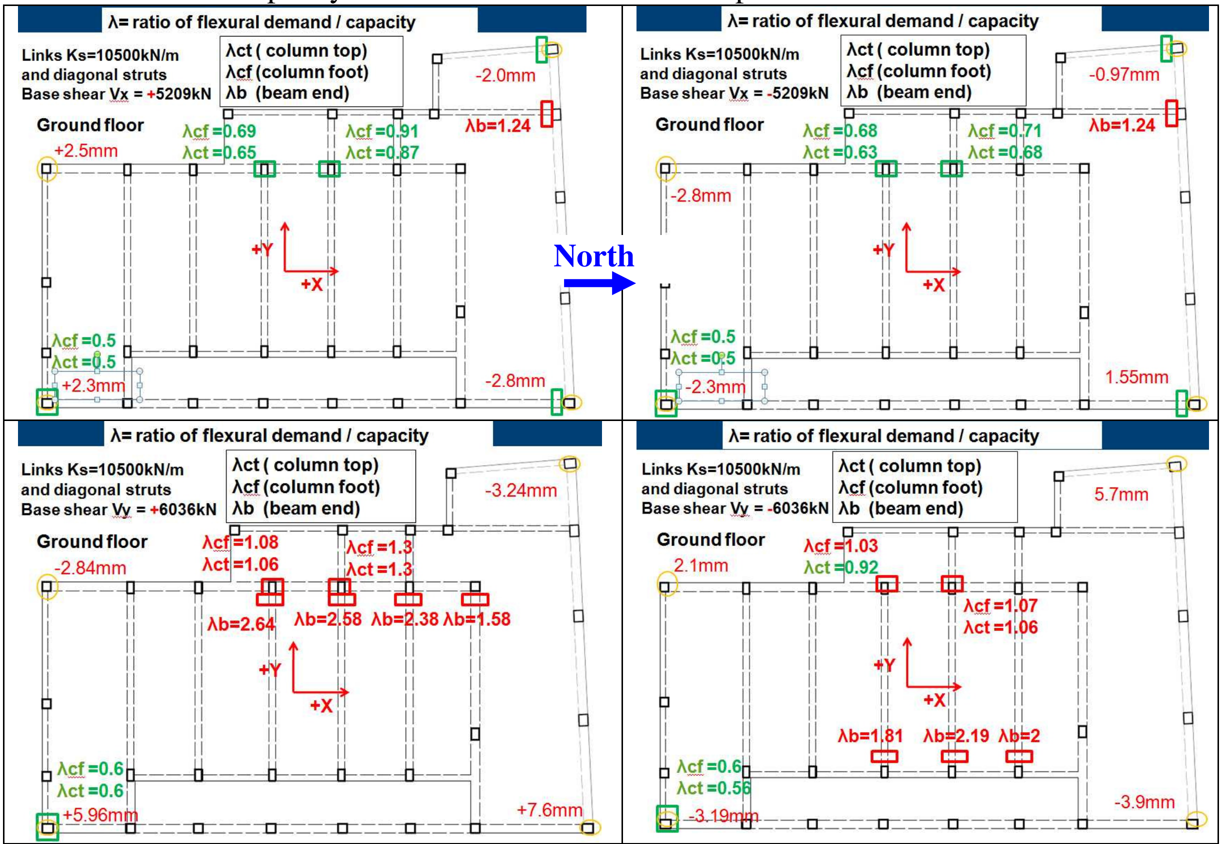
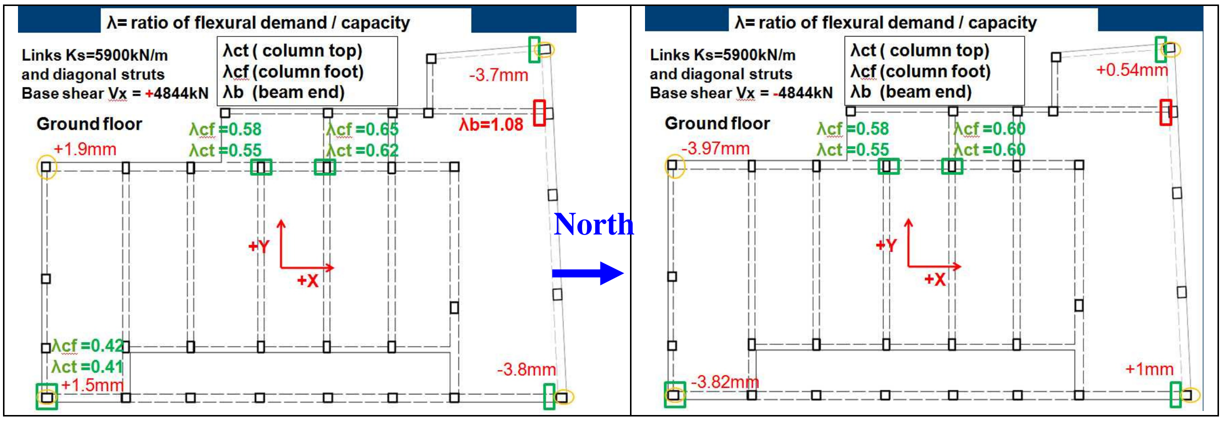


Figure 22. Flexural demand/capacity ratio values of R/C elements at the ground floor. Structure with diagonal struts and medium soil conditions.



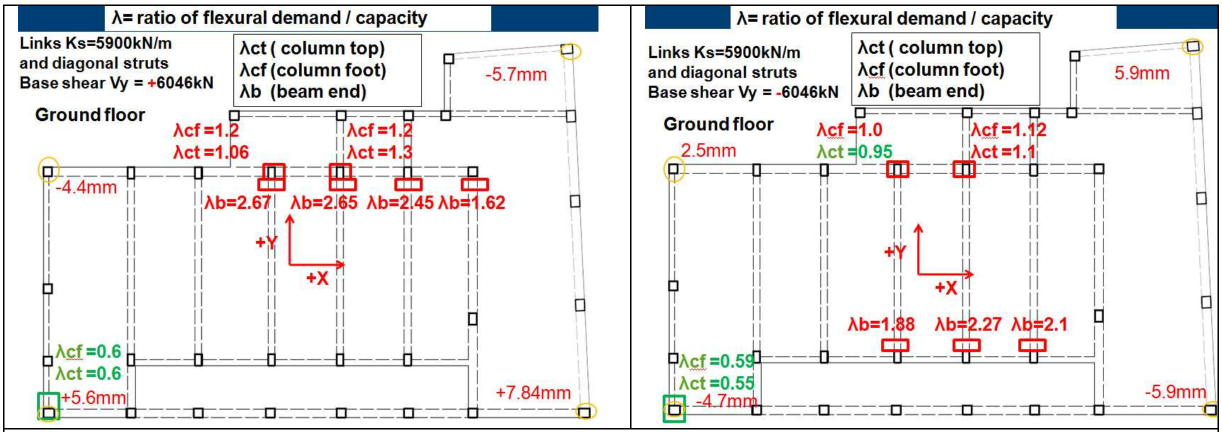


Figure 23. Flexural demand/capacity ratio values of R/C elements at the ground floor. Structure with diagonal struts and flexible soil conditions.

These **λ** ratio values are further detailed as **λb** when this ratio value refers to the end of a beam; when the ratio value refers to the top of a column is designated as **λct** or **λcf** when it refers to the foot of the column. When these ratios attain values larger than one (**λ > 1**, e.g. the predicted demand is larger than the capacity) then structural damage is expected to develop at this location. When this is the case the relevant value is plotted with the red color in figures 21, 22 and 23 otherwise is plotted with the green color when these ratio values are smaller than one (**λ < 1**), indicating that no structural damage is expected. Selected results are presented in the plots of figures 21, 22 and 23 for the columns of the ground floor and for the beams of the ground floor ceiling. Apart from these **λ** ratio values, the displacement demands at the four corners of the ground floor ceiling are also indicated in each plot of figures 21 to 23. The following remarks can be made on the basis of these results.

1. The largest **λct** or **λcf** (demand/capacity) ratio values are observed for the columns of the ground floor in the case of rigid soil conditions. The corresponding values for the medium or flexible soil conditions are smaller in comparison and many times become smaller than 1. This indicates that whereby structural damage is predicted in the columns for the rigid soil the presence of flexible soil results in most cases that these columns remain undamaged. The above observation can be explained by the value of the corresponding level of seismic forces used in each case in terms of base shear amplitude.
2. For approximately the same level of base shear amplitude in either the ±North-South (x-x) or ±East-West (y-y) direction the predicted performance of the ground floor columns demonstrates that the studied structure is more vulnerable when the seismic forces are applied in ±the East-West (y-y) direction than when applied in the ±North-South direction. This must be partly explained by the difference in the presence of the masonry infills as well as by the fact that the structural system in the ±the East-West (y-y) direction is composed by a series of two-column two-story R/C one-bay frames that, as already mention, have a clear span of 8.40m and develop high flexural demands.
3. For the same reasons, the beams of these frames are predicted to have large flexural demands that exceed in many cases the corresponding capacities (**λb > 1**), thus predicting structural damage in many end of these beam locations when the seismic forces are applied in the ±the East-West (y-y) direction.
4. The largest value of the **λcf** ratio (demand/capacity at the foot of the ground floor columns) is predicted for the column at the South-East corner at the ground floor of the City Hall building for the case of the rigid soil (**λcf** =2.12, figure 21). This occurs when the seismic forces are applied towards the North. The same column is predicted to be in

distress for the same soil conditions when the direction of the seismic forces is towards the West. This prediction agrees, up to a point, with the in-situ observations as this column exhibited signs of distress (figures 24 to 26).

5. Another location that the numerical analysis results predict column structural damage is the one at the west foot of the 4th from the left long y-y R/C frame with a clear span of 8.40. The value of the **λcf** ratio (demand/capacity at the foot of the ground floor columns) attains its largest value for rigid soil and seismic forces applied towards the West (**λcf** =1.42, figure 21). Structural damage is also predicted for this column for the same direction of seismic forces even for medium and flexible soil conditions; this time, however, the value of this ration is lower than the value for rigid soil (**λcf** =1.30 for medium soil figure 22 and **λcf** =1.20 for flexible soil figure 23). It should be pointed out here that this is also a location that is quite close to the accelerograph (figure 21) that recorded the seismic ground motion which was made use extensively here (see also figures 4 to 6). This prediction also agrees, up to a point, with in-situ observations as this column exhibited signs of distress (figures 4 to 6).
6. Structural damage is also predicted at the ends of the ground floor beams that belong to the long y-y R/C frame with a clear span of 8.40.This is predicted for all soil conditions when the seismic forces are applied in the East-West (y-y) direction. The largest **λb** ratio value (flexural demand/capacity for the beams) is obtained for the case of the rigid soil and the seismic forces applied towards the West (**λb**=2.28, figure 21). In-situ observations could not confirm obvious structural damage and distress at these locations. This is partly due to the fact that heavy plaster covered these locations which were at a considerable height from ground level as not to be easily accessible.

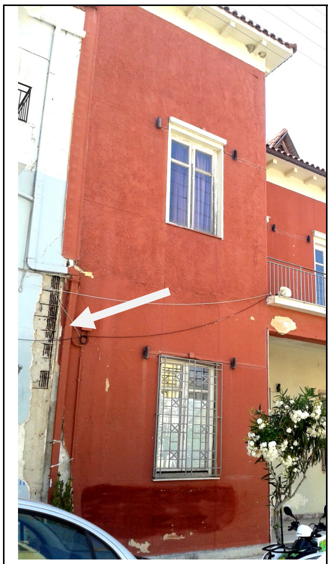


Figure 24. Spalling of concrete all along the height of this column

Figure 25. The exposed corroded longitudinal reinforcement at the foot of this column.

Figure 26. Measuring in-situ the distance between the stirrups of this column

6.2 Results from the Push-over numerical analyses with non-linear mechanisms for the two-node links at the foundation-soil interface, for the diagonal struts and all the R/C members

As already mentioned, in this 2nd category of numerical seismic analyses, the non-linear behaviour of all the R/C members is also considered here. A considerable number of numerical simulations were performed. Here only summary results are presented in figure 27.

The base shear (Q) versus horizontal displacement (δ) at the center of mass of the 1st story ceiling response is depicted in this figure. At the same time for certain levels of the base shear amplitude the numerically predicted non-linear mechanisms that develop at this stage are also indicated in this figure as follows:

a) At the level of a base shear approximately equal to 7000KN the formation of plastic hinges is predicted for the foot of the ground floor columns together with the yielding of the masonry infills at the South side.

b) At the level of a base shear approximately equal to 9500KN the yielding of the North side masonry infills is predicted. This is followed by the yielding of the interior masonry infills at the level of base shear approximately equal to 14000KN. The later occurs for levels of horizontal displacements at the center of mass of the 1st story ceiling of the order of 50mm.

c) Further increase in this horizontal displacement level leads to relatively modest increase in the resistance of the structure in this East-West direction, in terms of base shear levels, with plastic hinges developing at the top and foot of all the columns of the ground floor together with large horizontal displacement amplitudes at the 1st story ceiling (120mm, ground floor collapse mechanism).

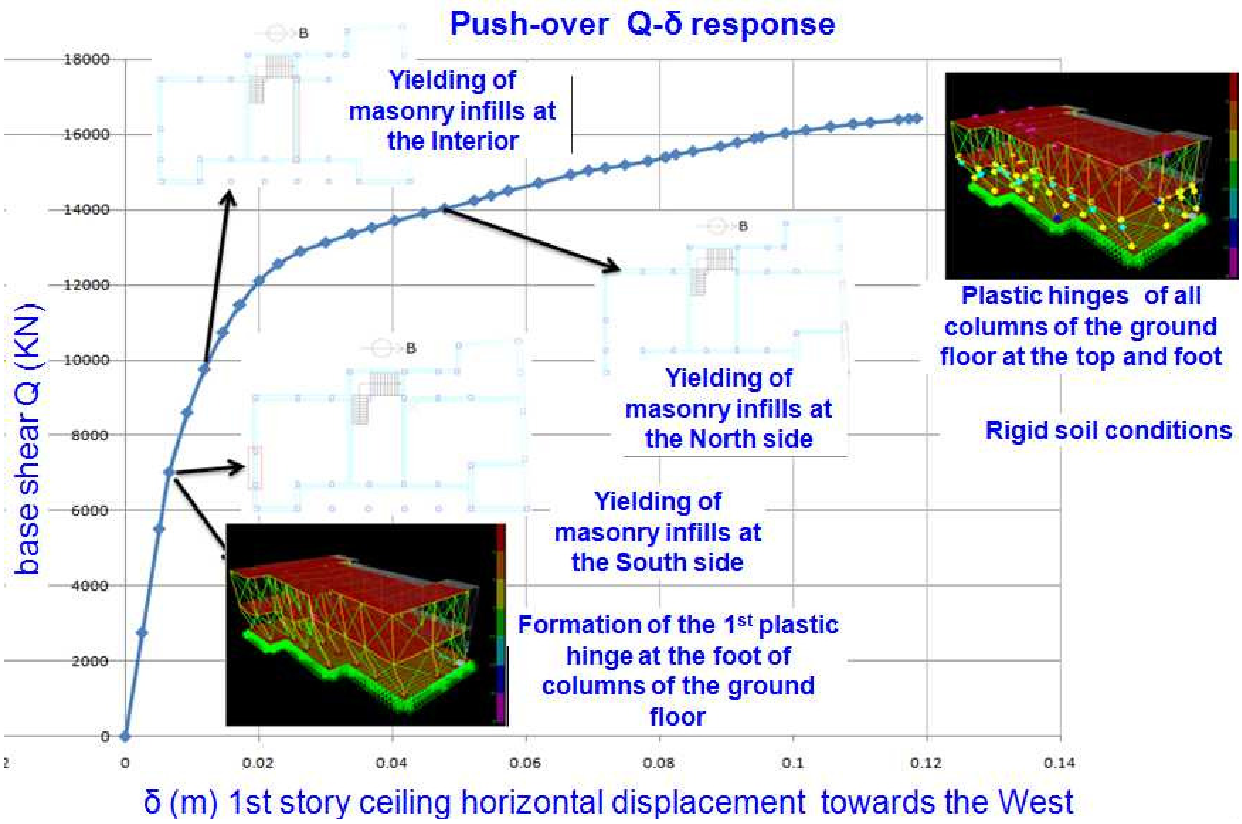


Figure 27. Push-over response of the City Hall structure with rigid soil conditions being forced towards the West.

As can be seen from the results of this analysis, the seismic force levels required to result in serious structural damage are above the forcing levels that were applied in the 1st category of numerical analyses presented previously.

One observation that becomes apparent from this comparison rises from the fact that both the 1st and 2nd category of the numerical analyses followed here do no incorporate the loss of bearing capacity of the masonry infills after a certain inter-story drift limit. This limitation can be tackled in a valid way by applying the full non-linear methodology described in [10]. Therefore, although these 2nd category numerical predictions can be accepted in describing reasonably well in a qualitative way all the incorporated various non-linear mechanisms, such as masonry infills, flexural plastic hinges of the R/C members, tension cut-off for the soil-foundation interface, the predicted seismic resistance in terms of base shear levels can be considered as rather non-conservative. In any case, the results of this category of numerical analyses demonstrate once more the contribution of well built masonry infills to resisting the seismic forces up to the point of their failure. This is more likely to occur when the earthquake excitation is of relatively short duration, as was the case of the strongest aftershock of 3rd of February, 2015 of the Kefalonia earthquake sequence that is studied here.

7 CONCLUSIONS

- It was demonstrated that masonry infills, as is already known, can significantly influence the seismic response of an R/C framed structural system by increasing its stiffness.

- It was also demonstrated that in certain cases these masonry infills can also increase the seismic capacity of such structural systems, when adverse effects of short columns or soft stories are avoided in these structural systems.
- The numerical predictions of the 1st category of analyses, apart from the above conclusion, can be considered as rather realistic, as these predictions are in reasonably good agreement with corresponding signs of distress of the ground floor columns of the Lixouri City Hall observed after this earthquake.
- The 2nd category of analyses result in base shear versus horizontal displacement numerical predictions that describe reasonably well the various non-linear mechanisms in a qualitative way. The results of this category also underline the favourable contribution of well built masonry infills of the Lixouri City Hall in resisting the seismic forces up to the point of their failure. This is likely to occur when the earthquake excitation is of relatively short duration, as was the case of the strongest aftershock of 3rd of February, 2015 of the Kefalonia earthquake sequence that is studied here.
- One observation that became apparent from the comparison of the numerical results with the actual performance rises from the fact that both the 1st and 2nd category of the numerical analyses followed here do no incorporate the loss of bearing capacity of the masonry infills after a certain inter-story drift limit. This limitation can be tackled in a valid way by applying the full non-linear methodology that is presented in [10].
- It was shown, that the seismic force levels which were assumed to have acted on the City Hall building are well beyond those that were valid at the time of the design and construction of this building. It was also shown that the seismic force levels that this structure as well as many neighboring structures experienced during the 3rd of February earthquake are higher than the force levels defined by the current provisions of Eurocode part 8.
- Factors that may have contributed to the relatively limited structural damage for these high seismic force levels, apart from the contribution of well built masonry infills, is the influence of the soil deformability that through the effects of the radiation damping may have had favourable effects in cases like the one examined here with a continuous foundation slab that prohibited the development of destructive differential foundation displacements and settlements during this earthquake excitation.

ACKNOWLEDGEMENTS

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